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FLOW THROUGH SUBMERGED RECTANGULAR ORIFICES WITH MODIFIED CONTRACTIONS[1]

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INTRODUCTION

The measurement of water flowing in open channels is a matter of growing importance, especially throughout the irrigated West, where the adoption of more economical methods of water delivery and canal management often is retarded by a lack of information concerning measuring devices adapted to specific field conditions. Where the use of free-flow weirs is prohibited by the low grade of canals and ditches, some type of submerged orifice has been substituted in many cases, though probably the majority of such ditches still are without any provision for measurement, except an occasional use of the current meter. The majority of the orifices installed have had complete end and bottom contractions, the choice being due, no doubt, to the more extensive and reliable information available concerning orifices of this type. The principal objections to a submerged orifice with complete contractions are the cost of the structure and the fact that it is not adapted to the measurement of water that carries sand and silt. These factors have prevented the installation of many measuring devices, and many have been installed where accumulations of sand and silt have rendered the measurements either questionable or worthless.

The complete suppression of the bottom contraction and the partial suppression of the end contractions will give a velocity of approach that will prevent sand and silt troubles in the orifice box, and will also lessen the cost of the structure. This is practically what has been done on many irrigation systems where lateral head gates and farmers' turnouts have been used directly as a means of measuring the flow. There are innumerable sizes, shapes, and conditions of setting such structures,

[1] The work upon which this paper is based was done in the hydraulic laboratory, Fort Collins, Colo., under cooperative agreements between the Office of Experiment Stations and the Office of Public Roads and Rural Engineering, United States Department of Agriculture, and the Colorado Experiment Station.

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there often being several kinds on a single canal system; but, as few of them have been calibrated, their value as measuring devices is not great. Of the few experiments made upon submerged orifices with modified contractions, only a small percentage are comparable to irrigation conditions.

A series of experiments was made in the hydraulic laboratory at Fort Collins, Colo., during the summer of 1914, for the purpose of developing some form of submerged orifice that would meet practical conditions.[1] It was practically impossible to make experiments upon all the different arrangements of structures used in the field, because of their infinite number and the further fact that many of them are essentially not adapted

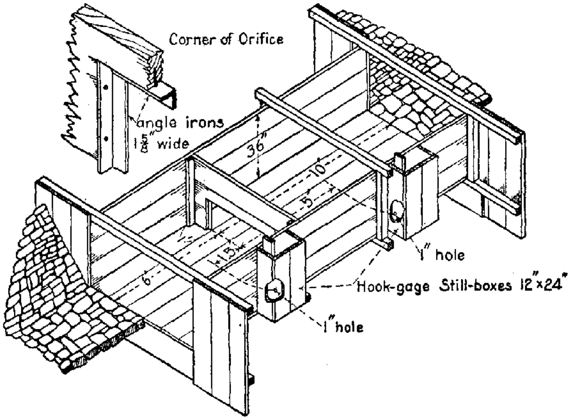

FIG. 1.—Standard box for submerged rectangular orifices with modified contractions.

to the measurement of water with a reasonable accuracy. Obviously it was necessary to work toward a standardization of the dimensions and arrangement of the orifice and orifice box in order that it might meet as nearly as possible the many conditions under which it would have to be installed in the field and still give a discharge to conform to a general formula or table. Several series of experiments were made before a set of conditions was decided upon as the standard.

The standard is a simple arrangement of orifice without bottom contraction but with angle-iron sides and top, and with end contractions of 1 foot (fig. 1). In this form it is a measuring device exclusively, but it may be combined with a gate, as indicated in Table I and the

[1] For a description of the hydraulic laboratory, see the following:

Cone, V. M. Hydraulic laboratory for irrigation investigations, Fort Collins, Colo. *In* Engin. News, v. 70, no. 14, p. 662–665, 5 fig. 1913.

—— Flow through weir notches with thin edges and full contraction. *In* Jour. Agr. Research, v. 5, no. 23, p. 1051–1113, 21 fig. 1916.

accompanying text figures, and a corresponding correction applied to the discharge table. The corrections are given for several different arrangements of metal and of wood gate guides, metal and wood edges of the orifice, and with and without a bottom contraction strip such as often is used for a bottom gate stop. It is expected that with these data many engineers and canal managers can arrange structures to meet their local demands and still give a reasonably dependable measurement of the quantity of water passing through the orifices. If the orifice box is built of concrete, the cross wall in which the orifice is placed must be given a flaring enlargement downstream from the orifice, to allow lateral expansion of the issuing stream of water.

Although the submerged orifice is a means to a satisfactory solution of some practical problems, it can not well be classed with precise measuring devices. Its discharge is influenced by comparatively trivial factors the identity of which often is unknown. Sand and silt troubles are eliminated by modifying the end and bottom contractions, but floating trash will accumulate in the orifice box. The new type of weir,[1] with suppressed bottom contraction, is entirely self-cleaning of trash, sand, and silt, and has practically the same accuracy as the modified orifice. It is therefore better to install the new weir where there is sufficient fall in the ditch to give free flow and where a separate headgate is provided.

Since there is a close relation between the accuracy of the measurement of flow and the accuracy of the determination of the head acting on the orifice, it is essential that some form of close-reading gage be used with the submerged orifice. The stilling well is necessary because of the comparatively high velocity of water in the orifice box.

ARRANGEMENT OF EXPERIMENTAL APPARATUS

The concrete weir box in the hydraulic laboratory is 6 feet deep, 10 feet wide, and 20 feet long, with a channel of approach about 60 feet long. By-passes in the side of the weir box permit a nice control of the water level. In an opening near the top and middle of the end wall of the weir box is placed a T-iron frame, 3 feet high by 6 feet long, in which the weir and orifice plates are placed for experimental purposes. In the series of experiments with orifices having modified contractions it was necessary to cover this opening with planks 1⅝ inches thick, made rigid and watertight. A floor of matched lumber was built in the concrete weir box about 3 feet above the bottom, as shown in figure 2. This floor was made level, rigid, and tight against the orifice bulkhead. Another floor, having a length of 6 feet, was placed on the downstream side of the orifice bulkhead, and made level with the floor on the upstream side.

The orifice was made by cutting a rectangular opening in the bulkhead so the bottom of the opening was at the floor line. The smallest orifice

[1] Cone, V. M. A new irrigation weir. *In* Jour. Agr. Research, v. 5, no. 24, p. 1127–1143, 16 fig. 1916.

was used first. Pieces of angle iron having a width of 1⅝ inches and a thickness of $\frac{3}{16}$ inch, with an edge planed square, or strips of wood 1⅝ inches square, were inserted in the opening to be flush with the upstream face of the bulkhead. These strips were screwed to the bulkhead to make the orifice opening true, of the desired size, and to prevent leakage. The condition of orifice taken as the standard had angle-iron sides and top, but no bottom contraction (fig. 3), though other experiments were made with a similar arrangement of wood strips and with a bottom contraction of 1⅝ inches formed by the width of the angle iron or wood strip. There were also several arrangements of metal and wood gate guides (see fig. 3 to 11, inclusive).

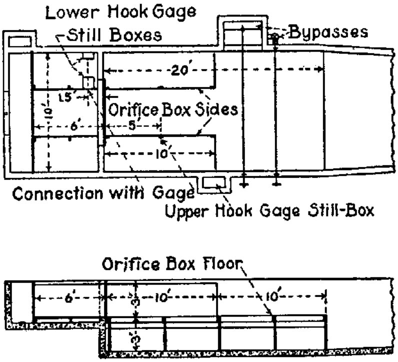

FIG. 2.—Plan, elevation, and section of orifice box in concrete channel.

A bulkhead placed across the channel, 6 feet downstream from the orifice bulkhead, contained a 20-inch square steel head gate the bottom of which was at the floor line. This gate was operated by a screw lift, which permitted a fair regulation of the water level downstream from the orifice; but the finer regulation was obtained by a 2-inch valve placed in the side of the channel.

Sections, 3 feet high and 10 feet long, made of matched lumber were used for the sides of the orifice box or channel of approach to the orifice. They could be moved to any position desired and fastened firmly to the floor. At the upstream end of the side sections, wings were attached at an angle of 90°, while the other end of the side sections butted against the orifice bulkhead. Although this box was practically water-tight, the question of leakage was of little importance because the box was entirely surrounded by water. Similar adjustable sides were placed downstream from the orifice and unless otherwise stated the widths of the channels of approach and recession were equal. In all the experiments the sides were set parallel and vertical and at an equal distance from the center line of the orifice.

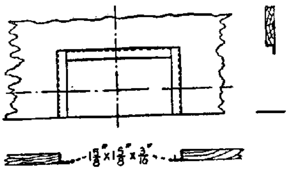

FIG. 3.—Elevation and sections of standard orifice without bottom contraction.

The head on the upstream side of the orifice was determined in the concrete still box by means of a standard type Boyden hook gage. The head was communicated to the still box through four lengths of ¾-inch hose connected to 1-inch pipe nipples placed through the side of the channel of approach until just flush with the inner face. These pipes were placed close together, so as to give an average distance of 5 feet from the plane of the orifice. The head on the upstream side of the orifice was kept constant by means of the by-passes and the main regulating gates at the storage reservoir.

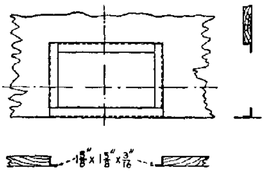

FIG. 4.—Elevation and sections of orifice with bottom contraction.

The head on the downstream side of the orifice was measured with a hook gage placed in a metal still box anchored to the concrete wall. The velocity of recession was so great in some cases as to cause a pulsation in the still box, which prevented a reasonably accurate measurement of the height of the water level. This was satisfactorily overcome by placing a metal tank, 12 by 12 by 30 inches, between the hook gage still box and the channel of recession. This regulating tank was connected with the channel of recession by a 1-inch pipe nipple placed through the side of the channel 1.5 feet from the plane of the orifice and 0.5 foot above the floor. The regulating tank was connected to the still box by a single piece of ¾-inch hose about 2 feet long.

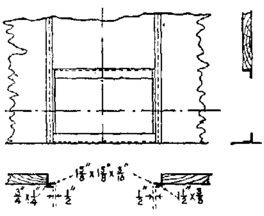

FIG. 5.—Elevation and sections of orifice with iron gate guides.

For each setting of orifice and arrangement of orifice box a number of observations, sufficient to determine the discharge curve, were made with different elevations of the water levels upstream and downstream from the orifice. The depth of water in the channel of approach remained constant for each set of observations, while the depth downstream was changed. However, no observation was started until the desired conditions of flow had been secured, and these conditions were not allowed to vary during the observation. The volume of water which flowed through the orifice during each test was determined accurately in the calibrated concrete tanks.

The exact dimensions of the orifices were measured with a micrometer caliper before and after the experiment, and, where slight changes were caused by swelling of the wood, average dimensions were taken. Usually

there was no appreciable change, but occasionally there was a change amounting to a few ten-thousandths of an inch.

In some of the experiments, where the greatest quantities of water passed through the orifice, there was a tendency toward vortexes. The surface of the water immediately upstream from the orifice would take a whirling motion with a greater or less depression, but the funnel never was sufficiently complete to allow air to be drawn through the orifice. The effect of these vortexes upon the discharge through the orifices was not apparent under the conditions of the experiments, but it might amount to considerable with a smaller depth of water in the channel of approach.

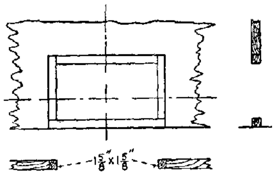

FIG. 6.—Elevation and sections of broad-edged orifice.

STANDARD CONDITIONS AND DIMENSIONS FOR SUBMERGED RECTANGULAR ORIFICES WITH MODIFIED CONTRACTIONS

As a result of experiments with several different arrangements the conditions described below and illustrated in figures 12 and 13 were taken as the standard because they appear to give the most reliable results, meet the practical demands for a measuring device of this type, and reduce the cost of construction. It is essential that the orifices and orifice boxes be built according to these specifications if the discharge formula or table is to be used. The influence of various changes in the size of the box and arrangement of the orifice is shown in Table I.

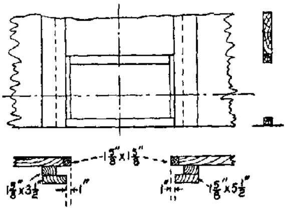

FIG. 7.—Elevation and sections of broad-edged orifice with wood gate guides.

The total length of the orifice box is 16 feet, 10 feet of which forms the channel of approach. Wings set at an angle of 90° are attached to the sides of the upstream end of the orifice box. The floor of the box is level throughout and at the same elevation as the bottom of the canal. The box should be set in the center line of the canal, so as to allow the water to enter the box in straight lines. The sides are parallel and are placed apart a distance equal to the length of the orifice plus 2 feet. Orifices of all sizes have end-contraction distances of 1 foot.

The orifice must have sharp sides and top, and no bottom contraction. Angle irons 1⅝ inches wide were used in the experiments and were placed as shown in figure 3. The orifice must be placed with its greatest dimen-

sion horizontal. If it is desirable to use an orifice with bottom contraction, or with wood sides and top, or with gate guides and gate, the discharge tables may be corrected in accordance with the data given in Table I and the deductions from that table given on pages 106 to 108.

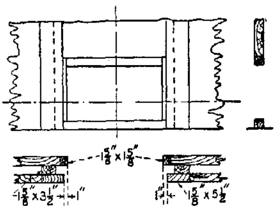

FIG. 8.—Elevation and sections of broad-edged orifice with wood gate guides and wood backing.

The elevations of the water levels in the channels of approach and recession should be taken in separate stilling boxes, one connection being 5 feet upstream and the other 1.5 feet downstream from the plane of the orifice. The connections should be through the side of the orifice box about 0.5 foot above the floor line.

The discharge tables were computed for a depth of 2.5 feet of water in the channel of approach. This depth was used in nearly all the standard experiments, the exceptions being with some of the smaller orifices, where a depth of 2.75 feet was used. This slight difference was not sufficient to change the discharge appreciably, because the velocity of approach was small in both cases.

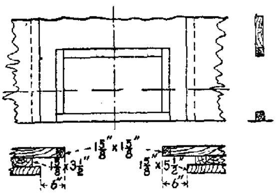

FIG. 9.—Elevation and sections of broad-edged orifice with wood gate guides moved out 6 inches.

Table I contains a summary of the results of the 317 observations with 60 different combinations of sizes of orifices, sharp and thick edges, with and without gate guides, with and without small bottom contraction, with different depths of water in the channel of approach, and with different end contractions in the channel of approach and recession. The data in the table and the figures referred to in the column to the right of the equations indicate the conditions under which each set of observations was made, with the exception of No. 49 and 50. No. 49 was with the sides of the channel of approach set at a width of 10 feet and the sides of the channel of recession set at a width of 3.0 feet, which gave end contractions of 0.5 foot. No. 50 was with the sides of both the channels of approach and recession set at a width of 10 feet. In all other cases the sides of the channel of approach and recession were set at an equal width.

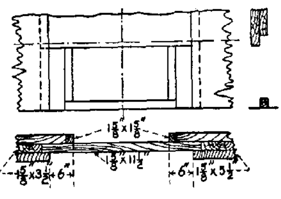

FIG. 10.—Elevation and sections of broad-edged orifice with wood gate slide and guides.

The words "with" or "without" in the column headed "Bottom contraction" indicate whether the floor formed the bottom of the orifice, or an angle iron or wood strip extended above the floor a distance of 1⅝ inches, such as would be used for a bottom gate stop.

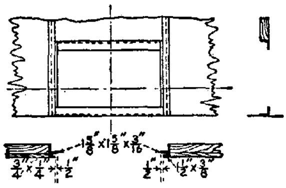

FIG. 11.—Elevation and sections of standard orifice with iron gate guides and backing.

In the column headed "Deviation from discharge table" are given the percentages by which the discharges computed from the individual formulas fail to agree with the discharges given in Table III, which were computed from the general formula for the standard conditions of orifices. The discharges computed from the general formula were taken as the basis of comparison, plus and minus signs representing greater and less discharges, respectively, by the individual formula than by the general formula. This arrangement allows the effects of various alterations in the size and setting of the orifice and orifice box to be compared more easily than would be possible from a number of complete discharge tables, and indicates the correction which should be applied to make the discharge tables applicable for each condition given.

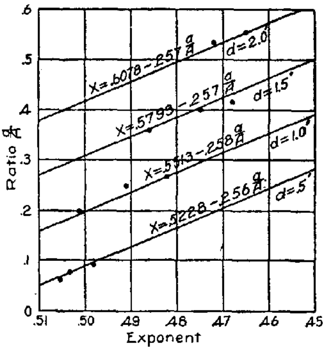

FIG. 12.—Plots of exponent values of equations in Table II

The equations given in Table I were obtained from large-scale logarithmic plots of the experimental data for each arrangement of orifice and orifice box by the use of the discharges and differences in heads as the coordinates. The straight-line curves represented the data quite accurately for all conditions except those given under No. 12 and 46 in Table I. In these two cases the discharge data for heads of 0.8 foot were greater by about 2 per cent than those represented by the curves, but all other points fell on the lines. It will be noted

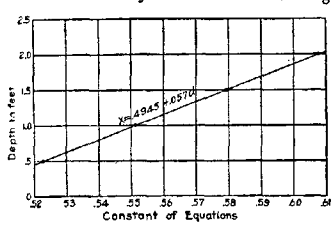

FIG. 13.—Plots of constants of equations given in figure 12.

that both exceptions were for high heads and for the same size of orifice, 0.5 by 2.0 feet, with and without angle-iron contraction in the bottom

of the orifice, and a duplication of the experimental work checked the results as given. No explanation is apparent for this isolated inconsistency.

TABLE I.—*Summary of results of experiments, showing the influence of various changes in the size of the box and the arrangement of the orifice*

Combination No.	Size of orifice.		Width of channel of approach.	Depth of water in channel of approach.	Bottom contraction.	Equation of discharge curve.	See text figure No.	Deviation from discharge table.			
	Depth.	Length.						Head.	Per cent.	Head.	Per cent.
	Ft.	*Ft.*	*Ft.*	*Ft.*				*Ft.*		*Ft.*	
1	0.5	1.0	2.0	1.700	Without..	$Q=0.604\ a\sqrt{2g}\ H^{0.4969}$......	2	0.03	+ 3.9	0.75	0.0
2	.5	1.5	2.5	1.750	...do.......	$Q=\ .606\ a\sqrt{2g}\ H^{.4934}$......	2	.03	+ 3.7	.75	+ 0.5
3	.5	2.0	3.0	1.750	...do.......	$Q=\ .606\ a\sqrt{2g}\ H^{.4935}$......	2	.03	+ 2.3	.75	+ 0.4
4	.5	1.0	2.0	2.700	...do.......	$Q=\ .602\ a\sqrt{2g}\ H^{.5062}$......	2	.03	+ 0.5	.75	− 0.5
5	.5	1.5	2.5	2.750	...do.......	$Q=\ .609\ a\sqrt{2g}\ H^{.4654}$......	2	.03	− 1.4	.75	+ 0.6
6	.5	2.0	3.0	2.750	...do.......	$Q=\ .605\ a\sqrt{2g}\ H^{.4978}$......	2	.03	+ 0.5	.75	+ 0.1
7	1.0	2.0	3.0	2.500	...do.......	$Q=\ .629\ a\sqrt{2g}\ H^{.4987}$......	2	.03	+ 3.8	.75	+ 2.3
8	1.0	3.0	4.0	2.500	...do.......	$Q=\ .631\ a\sqrt{2g}\ H^{.5054}$......	2	.03	− 2.2	.75	+ 1.8
9	1.0	4.0	5.0	2.500	...do.......	$Q=\ .627\ a\sqrt{2g}\ H^{.4881}$......	2	.03	+ 0.6	.75	+ 1.1
10	.5	1.0	3.0	1.750	...do.......	$Q=\ .602\ a\sqrt{2g}\ H^{.4975}$......	2	.03	+ 3.2	.75	− 0.5
11	.5	1.5	3.5	1.750	...do.......	$Q=\ .603\ a\sqrt{2g}\ H^{.5040}$......	2	.03	− 0.5	.75	− 0.2
12	.5	2.0	4.0	1.750	...do.......	$Q=\ .605\ a\sqrt{2g}\ H^{.4978}$......	2	.03	+ 0.4	.75	0.0
13	.5	1.0	3.0	2.750	...do.......	$Q=\ .605\ a\sqrt{2g}\ H^{.5003}$......	2	.03	+ 0.7	.75	− 0.2
14	.5	1.5	3.5	2.750	...do.......	$Q=\ .601\ a\sqrt{2g}\ H^{.5017}$......	2	.03	0.0	.75	− 0.4
15	.5	2.0	4.0	2.750	...do.......	$Q=\ .602\ a\sqrt{2g}\ H^{.5027}$......	2	.03	− 1.8	.75	− 0.6
16	1.0	2.0	4.0	2.500	...do.......	$Q=\ .614\ a\sqrt{2g}\ H^{.5005}$......	2	.03	+ 0.8	.75	− 0.1
17	1.0	3.0	5.0	2.500	...do.......	$Q=\ .611\ a\sqrt{2g}\ H^{.4876}$......	2	.03	+ 0.7	.75	− 1.0
18	1.0	4.0	6.0	2.500	...do.......	$Q=\ .617\ a\sqrt{2g}\ H^{.4845}$......	2	.03	+ 0.2	.75	− 0.4
19	1.5	3.0	5.0	2.500	...do.......	$Q=\ .644\ a\sqrt{2g}\ H^{.4950}$......	2	.03	− 0.1	.50	+ 1.0
20	1.5	4.0	6.0	2.500	...do.......	$Q=\ .639\ a\sqrt{2g}\ H^{.4806}$......	2	.03	+ 0.1	.40	0.0
21	1.5	4.5	6.5	2.500	...do.......	$Q=\ .638\ a\sqrt{2g}\ H^{.4788}$......	2	.03	− 1.1	.35	− 0.7
22	2.0	4.0	6.0	2.500	...do.......	$Q=\ .674\ a\sqrt{2g}\ H^{.4762}$......	2	.03	+ 1.6	.25	+ 1.6
23	2.0	4.5	6.5	2.500	...do.......	$Q=\ .692\ a\sqrt{2g}\ H^{.4842}$......	2	.03	− 0.8	.20	+ 1.7
24	.5	1.0	4.0	1.750	...do.......	$Q=\ .600\ a\sqrt{2g}\ H^{.4967}$......	2	.03	+ 3.2	.75	− 0.8
25	.5	1.0	4.0	2.750	...do.......	$Q=\ .599\ a\sqrt{2g}\ H^{.4977}$......	2	.03	+ 2.7	.75	− 1.0
26	1.5	3.0	6.0	2.500	...do.......	$Q=\ .623\ a\sqrt{2g}\ H^{.4850}$......	2	.03	+ 0.1	.50	− 1.6
27	.5	1.0	2.0	1.625	With......	$Q=\ .618\ a\sqrt{2g}\ H^{.4975}$......	3	.03	+ 6.1	.75	+ 2.2
28	.5	1.5	2.5	1.600	...do.......	$Q=\ .625\ a\sqrt{2g}\ H^{.5005}$......	3	.03	+ 4.0	.75	+ 3.2
29	.5	2.0	3.0	1.600	...do.......	$Q=\ .632\ a\sqrt{2g}\ H^{.5005}$......	3	.03	+ 3.9	.75	+ 4.5
30	.5	1.0	2.0	2.625	...do.......	$Q=\ .616\ a\sqrt{2g}\ H^{.5007}$......	3	.03	+ 2.7	.75	+ 1.6
31	.5	1.5	2.5	2.600	...do.......	$Q=\ .621\ a\sqrt{2g}\ H^{.5008}$......	3	.03	+ 3.2	.75	+ 2.5
32	.5	2.0	3.0	2.600	...do.......	$Q=\ .626\ a\sqrt{2g}\ H^{.4961}$......	3	.03	+ 3.9	.75	+ 3.6
33	1.0	2.0	3.0	2.500	...do.......	$Q=\ .642\ a\sqrt{2g}\ H^{.4908}$......	3	.03	+ 7.1	.75	+ 4.5
34	1.0	3.0	4.0	2.500	...do.......	$Q=\ .655\ a\sqrt{2g}\ H^{.4934}$......	3	.03	+ 5.1	.75	+ 6.0
35	.5	1.5	3.5	2.600	...do.......	$Q=\ .618\ a\sqrt{2g}\ H^{.5019}$......	3	.03	+ 2.4	.75	+ 2.0
36	.5	2.0	4.0	2.600	...do.......	$Q=\ .620\ a\sqrt{2g}\ H^{.4967}$......	3	.03	+ 2.6	.75	+ 2.5
37	1.0	2.0	4.0	2.500	...do.......	$Q=\ .632\ a\sqrt{2g}\ H^{.4977}$......	3	.03	+ 4.7	.75	+ 2.8
38	1.0	3.0	5.0	2.500	...do.......	$Q=\ .632\ a\sqrt{2g}\ H^{.4922}$......	3	.03	+ 6.2	.75	+ 2.6
39	1.0	4.0	6.0	2.500	...do.......	$Q=\ .635\ a\sqrt{2g}\ H^{.4887}$......	3	.03	+ 1.7	.75	+ 2.5
40	1.5	3.0	5.0	2.500	...do.......	$Q=\ .662\ a\sqrt{2g}\ H^{.4948}$......	3	.03	+ 3.0	.50	+ 4.0
41	1.5	4.0	6.0	2.500	...do.......	$Q=\ .656\ a\sqrt{2g}\ H^{.4947}$......	3	.03	+ 1.1	.40	+ 2.1
42	1.5	4.5	6.5	2.500	...do.......	$Q=\ .667\ a\sqrt{2g}\ H^{.4960}$......	3	.03	+ 0.9	.35	+ 3.2
43	2.0	4.0	6.0	2.500	...do.......	$Q=\ .697\ a\sqrt{2g}\ H^{.4916}$......	3	.03	+ 3.2	.25	+ 4.3
44	2.0	4.5	6.5	2.500	...do.......	$Q=\ .685\ a\sqrt{2g}\ H^{.4707}$......	3	.03	+ 2.9	.20	+ 2.9
45	.5	1.5	3.5	1.600	...do.......	$Q=\ .626\ a\sqrt{2g}\ H^{.5092}$......	3	.03	+ 1.1	.75	+ 3.1
46	.5	2.0	4.0	1.600	...do.......	$Q=\ .626\ a\sqrt{2g}\ H^{.4970}$......	3	.03	+ 4.3	.75	+ 3.6
47	.5	1.0	4.0	1.625	...do.......	$Q=\ .618\ a\sqrt{2g}\ H^{.5028}$......	3	.03	+ 4.2	.75	+ 2.0
48	.5	1.0	4.0	2.625	...do.......	$Q=\ .612\ a\sqrt{2g}\ H^{.4942}$......	3	.03	+ 4.4	.75	+ 1.2
49	.5	2.0	10.0	2.600	...do.......	$Q=\ .619\ a\sqrt{2g}\ H^{.4993}$......	3	.03	+ 2.3	.75	+ 2.4
50	.5	2.0	10.0	2.600	...do.......	$Q=\ .617\ a\sqrt{2g}\ H^{.5030}$......	3	.03	+ 0.9	.75	+ 1.9

TABLE I.—*Summary of results of experiments, showing the influence of various changes in the size of the box and the arrangement of the orifice*—Continued

Combination No.	Size of orifice.		Width of channel of approach.	Depth of water in channel of approach.	Bottom contraction.	Equation of discharge curve.	See text figure No.	Deviation from discharge table.			
	Depth.	Length.						Head.	Per cent.	Head.	Per cent.
	Ft.	*Ft.*	*Ft.*	*Ft.*				*Ft.*		*Ft.*	
51	1.0	2.0	4.0	2.500	With......	$Q=0.665\, a \sqrt{2g}\, H^{0.5019}$......	4	0.03	+ 8.6	0.75	+ 8.1
52	1.0	2.0	4.0	2.500	...do......	$Q=.637\, a \sqrt{2g}\, H^{.4974}$......	5	.03	+ 5.6	.75	+ 3.6
53	1.0	2.0	4.0	2.500	...do......	$Q=.694\, a \sqrt{2g}\, H^{.4939}$......	6	.03	+16.0	.75	+13.0
54	1.0	2.0	4.0	2.500	...do......	$Q=.694\, a \sqrt{2g}\, H^{.4963}$......	7	.03	+15.7	.75	+12.8
55	1.0	2.0	4.0	2.500	...do......	$Q=.643\, a \sqrt{2g}\, H^{.4977}$......	8	.03	+ 6.3	.75	+ 4.4
56	1.0	2.0	4.0	2.500	...do......	$Q=.669\, a \sqrt{2g}\, H^{.5025}$......	9	.03	+ 8.9	.75	+ 8.6
57	1.5	4.0	6.0	2.500	...do......	$Q=.674\, a \sqrt{2g}\, H^{.4839}$......	4	.03	+ 4.4	.40	+ 5.2
58	1.5	4.0	6.0	2.500	...do......	$Q=.669\, a \sqrt{2g}\, H^{.4800}$......	10	.03	+ 5.0	.40	+ 4.8
59	1.5	4.0	6.0	2.500	...do......	$Q=.659\, a \sqrt{2g}\, H^{.4798}$......	5	.03	+ 3.9	.40	+ 3.3
60	1.5	4.0	6.0	2.500	...do......	$Q=.722\, a \sqrt{2g}\, H^{.4898}$......	6	.03	+ 9.6	.40	+11.9

Although constant care was used in making, setting, and calibrating the orifices, placing the sides and bottom of the orifice box, and observing precautions to eliminate all known sources of error, still there are a few inconsistencies, or what appear to be inconsistencies, in the data, though a more complete understanding of the flow through orifices of this type may show them to be due to more or less similar influences. Since the experimental data made straight-line logarithmic plots, only a few points were necessary to define those lines within a comparatively small percentage of error. Most of the curves for comparable conditions are practically parallel, but in two cases the curves cross. No. 8 in Table I crosses No. 17, and No. 5 crosses No. 14, these lines representing the data very faithfully.

The experimental conditions for the greater differences of head and for the longer orifices were less reliable than for the smaller discharges, but the general agreement of the data indicates that the accuracy was within practical demands at least. The arrangement of the control gate at the end of the channel of recession very probably produced a backlash, which influenced the discharge, especially when the velocity of the water was great. The velocity of the water and cross currents also may have affected the hook-gage readings, but Table IV proves the average accuracy to be satisfactory. (See additional information given on p. 114.)

DEDUCTIONS FROM TABLE I

From an inspection of the coefficient and exponent values of the equations in Table I, the following general statements may be made:

The exponent decreases as the length of the orifice L increases so long as the depth of the orifice d and the cross-sectional area of the water in the channel of approach A remain constant. Although no two sets of experiments were made with exactly the same A, some are sufficiently close for purposes of comparison.

The exponent decreases as the ratio of the area of the orifice to the wetted cross-sectional area of the channel of approach $\frac{a}{A}$ increases.

The coefficient increases with an increase in the depth of the orifice *d*.

The coefficient increases with an increase in the area of the orifice *a*, but does not increase regularly when the area of the orifice *a* has been divided out of the aggregate coefficient value.

Although the end-contraction distance has been taken as 1 foot for all sizes of orifices as a standard condition, a comparison of the deviation of discharges from the standard formula given in Table III indicates that little error would be introduced by making the contraction distance 0.5 foot for the smaller sizes of orifices. Increasing the distance to 1.5 feet would cause a greater error.

No. 13 to 23, inclusive, are for standard conditions. A comparison of these data with the data for other experiments, shows that lessening the depth of water in the channel of approach approximately 1 foot increases the discharge approximately 2 or 3 per cent. The increase is much greater for low heads than for high heads, especially with the smaller orifices. This action is difficult to explain, but it may be due to the existence of a critical velocity below which the velocity of approach has only a moderate effect upon the discharge and above which the effect may be somewhat overcome by increased friction and eddy currents.

The addition of an angle-iron bottom contraction and iron gate guides (No. 51) increases the discharge about 8 per cent.

No. 58 was an experiment to determine the effect upon the discharge caused by the iron gate guides projecting from the plane of the orifice as a comparatively narrow strip. A comparison of No. 58 and 57 shows that filling out the bulkhead until the face was flush with the edge of the gate guide (fig. 11) made practically no change in the discharge.

An orifice made of wood about 1⅝ inches thick, with a bottom contraction of the same material about 1⅝ inches high (No. 57), increased the discharge 3.3 per cent for the low head and 5.6 per cent for the high head. A comparison of this increase with that due to the angle-iron bottom contraction added to the standard condition of orifice indicates that the substitution of wood 1⅝ inches thick in place of the angle iron in the standard orifices will make the discharge about 2 per cent greater than that given in the standard discharge table. It will be observed that there would be no bottom contraction with this condition.

A wood orifice 1⅝ inches thick, with wood gate guides (fig. 7), will give a discharge from 9.6 to 16 per cent greater than that of the standard table, and the increase is the greatest for the smaller orifices (see No. 53 and 60). This increase probably is due to the guides being nearly the same distance apart as the length of the orifice, but set back from the plane of the orifice far enough to make the action similar to the flow through a short pipe.

No. 53 and 54 and figures 7 and 8 show the projection of the wood gate guides from the bulkhead to have little effect upon the discharge.

In No. 55 (fig. 9) the wood gate guides were set back a distance of 0.5 foot, and the resulting discharge was from 4.4 per cent to 6.3 per cent greater than the standard. A further comparison of the conditions shown in figures 7 and 9 indicate the discharge with the gate guides set back 0.5 foot to be only about 0.5 per cent greater than that for a plain wood orifice without any gate guides.

A wood orifice, with gate guides set back 0.5 foot and with a wood gate slide, as shown in No. 56 (fig. 10), gave a discharge about 8.6 to 8.9 per cent greater than the standard. A comparison of No. 56 with No. 55 indicates that the increase due to gate slide alone is from 3 to 4 per cent.

Complete end contractions on the upstream side of the orifice, and 0.5, foot end contractions on the downstream side, with a bottom contraction of 1⅝ inches (No. 49), gave a discharge from 2.3 to 2.4 per cent greater than that of the standard, and this was about the average increase due to the bottom angle-iron contraction with the standard orifice box. Therefore there was apparently little effect due to the complete end contraction in the channel of approach; but the decrease which, theoretically, should have resulted may have been counterbalanced by the increased velocity of approach caused by the smaller end contraction in the channel of recession.

Complete end contractions, both upstream and downstream from the orifice, but with an angle-iron bottom contraction of 1⅝ inches (No. 50), caused a deviation from the standard discharge of −0.5 per cent for the low head and +2.9 per cent for the high head. The discharge for the high head under this condition was therefore about the same as the standard size of orifice box with the angle-iron bottom contraction, but the increase in end contractions caused a decrease in the discharge for the low head of 2 or 3 per cent. The discharge curves represent the experimental data very accurately, and there is no apparent reason for the failure to decrease the discharge on the higher heads.

From the equations for No. 4 to 9 and 13 to 23, inclusive, it will be seen that, for a constant depth of water in the channel of approach, and for a constant depth of orifice, but for different lengths of orifices, where there are three lines in a set, the exponent value is the greatest for the middle length. A plot of the three points makes the curve apparent, even though the numerical values do not indicate it. The reverse of this curve is true for the coefficient values, as is shown by the several conditions of contraction, when comparable conditions are inspected. A similar comparison for No. 35 to 44, which are with bottom angle-iron contraction, shows both the coefficient and exponent value for the middle length to be the lowest. Therefore the insertion of the angle-iron bottom contraction seems to have reversed the curve for the law of the exponents, but produced no change in the curve for the coefficients.

DERIVATION OF FORMULA FOR MODIFIED RECTANGULAR ORIFICES

Several unsuccessful attempts were made to derive a simple and accurate expression of the variation of the exponent and coefficient values in the individual equations No. 13 to 23, inclusive. As has been previously mentioned, the exponents and coefficients plot as a series of disconnected curves with the depths and lengths of orifices as the governing factors, apparently. The difficulty was experienced in determining just what factors should be used in an expression of the law of variation; and, though the following form is sufficiently accurate for practical purposes, it does not faithfully represent the variation. A more exact expression could have been obtained by using the same factors and expressing the variations as curves instead of straight lines, but the resulting formula would have been so complicated as to make it of doubtful practical value. It will be observed that the discharge formula appears to be more complicated than it really is, and the influence of velocity of approach has been expressed in terms of the wetted area of the channel of approach, which may be determined more easily, because there is a standard size of box for each length of orifice.

The experimental discharge data for the standard conditions of orifices and orifice boxes were plotted logarithmically against the areas of the orifices. The resulting series of curves were for each constant difference in head. From these curves the smoothed or balanced discharge values were taken and plotted logarithmically against the difference in heads. The equations of the average straight-line curves drawn through these points are given in Table II. The exponents in these equations were plotted against the ratio of the area of the orifice in square feet to the area of the wetted cross section of the channel of approach, $\frac{a}{A}$, as shown in figure 12. As previously noted, the exponents are in groups for the several areas of orifices with the same depth of orifice, and each group forms a detached curve which is probably parabolic in shape. To avoid a very complicated expression of their law of variation, they were assumed to be represented by straight lines which were parallel and had equal intercepts on the Y axis. The equation of each individual line is given in figure 12. The constants in these equations were plotted against the depths of the orifices (fig. 13), and the equation of the resulting curve was obtained. The substitution of this value in the equations given in figure 12 and with an average value for the coefficient of the ratio $\frac{a}{A}$, gave $n=0.4945+0.057d-\frac{0.257a}{A}$ as the general expression of the exponent of the head.

TABLE II.—*Equations of balanced discharge curves used in development of general formula*

Size of orifice.		Equation.
Depth.	Length.	
Feet.	*Feet.*	
0.5	1.0	$Q=2.424h^{0.5054}$.
.5	1.5	$Q=3.624h^{0.5038}$.
.5	2.0	$Q=4.822h^{0.4982}$.
1.0	2.0	$Q=9.858h^{0.5013}$.
1.0	3.0	$Q=14.842h^{0.4910}$.
1.0	4.0	$Q=19.779h^{0.4822}$.
1.5	3.0	$Q=22.751h^{0.4860}$.
1.5	4.0	$Q=30.374h^{0.4750}$.
1.5	4.5	$Q=34.293h^{0.4680}$.
2.0	4.0	$Q=44.668h^{0.4840}$.
2.0	4.5	$Q=48.600h^{0.4650}$.

The coefficients of the revised equations given in Table II were plotted logarithmically against the areas of the orifices, and they also were in separate groups for each depth of orifice, but are not shown here because the reduction in the size of the plot would obscure the grouping. Straight lines, drawn to meet at a common point, fairly represent the several sets of plotted points, and give a simple law of their variation. The equations of these lines follow:

When $d=0.5$ foot, $c=4.85a^{1.00}$.
$d=1.0$ foot, $c=4.90a^{1.01}$.
$d=1.5$ feet, $c=4.95a^{1.02}$.
$d=2.0$ feet, $c=5.00a^{1.03}$.

The coefficient and exponent values of the area of the orifice, a, were plotted and found to be represented by $(4.8+0.1d)$ and $(0.02d+0.99)$, respectively, which unite as the coefficient of the head $c=(4.8+0.1d)a^{(0.02d+0.99)}$.

Consolidating the expressions for the exponent and coefficient values of the head, h, gives the general formula for the discharge through submerged rectangular orifices placed according to the conditions which have been taken as the standard:

$$Q=\left((4.8+0.1d)a^{(0.02d+0.99)}\right)h^{(0.495d+0.06d-\frac{7.36a)}{A}}$$

in which "Q" equals the discharge in second-feet; "d" equals depth of orifice in feet; "a" equals area of orifice in square feet; "A" equals area of cross section of water in channel of approach in square feet; and "h" equals the difference in feet between the water levels upstream and downstream from the orifice.

TABLE III.—*Discharges through modified submerged rectangular orifices as computed from the formula*

Head in feet.	Head in inches.	Discharge (cubic feet per second).										
		0.5X1.0	0.5X1.5	0.5X2.0	1.0X2.0	1.0X3.0	1.0X4.0	1.5X3.0	1.5X4.0	1.5X4.5	3.0X4.0	2.0X4.5
0.03	0 3/8	0.408	0.624	0.843	1.69	2.64	3.61	4.10	5.70	6.52	8.02	9.22
.04	0 1/2	.473	.721	.973	1.95	3.04	4.15	4.73	6.55	7.48	9.20	10.55
.05	0 5/8	.529	.806	1.09	2.19	3.39	4.63	5.27	7.29	8.32	10.23	11.72
.06	0 3/4	.581	.884	1.19	2.40	3.71	5.06	5.77	7.95	9.07	11.16	12.77
.07	0 13/16	.628	.955	1.29	2.59	4.01	5.46	6.22	8.57	9.76	12.01	13.74
.08	0 15/16	.672	1.02	1.38	2.77	4.28	5.82	6.64	9.14	10.41	12.79	14.63
.09	1 1/16	.714	1.08	1.46	2.94	4.53	6.17	7.04	9.67	11.01	13.53	15.46
.10	1 3/16	.753	1.14	1.54	3.10	4.78	6.49	7.41	10.17	11.57	14.23	16.25
.11	1 5/16	.790	1.20	1.61	3.25	5.01	6.80	7.77	10.65	12.11	14.89	17.00
.12	1 7/16	.826	1.25	1.68	3.40	5.23	7.09	8.11	11.10	12.63	15.52	17.71
.13	1 9/16	.860	1.30	1.75	3.54	5.44	7.37	8.43	11.54	13.12	16.12	18.39
.14	1 11/16	.893	1.35	1.82	3.67	5.64	7.64	8.74	11.96	13.59	16.70	19.04
.15	1 13/16	.925	1.40	1.88	3.80	5.83	7.90	9.04	12.36	14.04	17.26	19.67
.16	1 15/16	.956	1.45	1.94	3.93	6.02	8.16	9.33	12.75	14.48	17.80	20.28
.17	2 1/16	.986	1.49	2.00	4.05	6.20	8.40	9.62	13.13	14.91	18.32	20.86
.18	2 3/16	1.01	1.54	2.06	4.17	6.38	8.64	9.89	13.49	15.32	18.82	21.43
.19	2 1/4	1.04	1.58	2.12	4.28	6.55	8.87	10.16	13.85	15.72	19.31	21.98
.20	2 3/8	1.07	1.62	2.17	4.39	6.72	9.09	10.42	14.19	16.11	19.79	22.52
.21	2 1/2	1.10	1.66	2.23	4.50	6.89	9.31	10.67	14.53	16.49	20.26	......
.22	2 5/8	1.12	1.70	2.28	4.61	7.04	9.52	10.92	14.86	16.86	20.71	......
.23	2 3/4	1.15	1.74	2.33	4.71	7.20	9.73	11.16	15.18	17.22	21.15	......
.24	2 7/8	1.17	1.77	2.38	4.81	7.35	9.93	11.39	15.50	17.57	21.58	......
.25	3 0	1.20	1.81	2.43	4.91	7.50	10.13	11.62	15.80	17.92	22.01	......
.26	3 1/8	1.22	1.85	2.48	5.01	7.65	10.33	11.85	16.10	18.26	......	......
.27	3 1/4	1.25	1.88	2.52	5.11	7.79	10.52	12.07	16.40	18.59	......	......
.28	3 3/8	1.27	1.92	2.57	5.20	7.93	10.71	12.29	16.69	18.91	......	......
.29	3 1/2	1.29	1.95	2.62	5.29	8.07	10.89	12.50	16.97	19.23	......	......
.30	3 5/8	1.32	1.99	2.66	5.38	8.21	11.07	12.71	17.25	19.55	......	......
.31	3 3/4	1.34	2.02	2.70	5.47	8.34	11.25	12.92	17.53	19.86	......	......
.32	3 13/16	1.36	2.05	2.75	5.56	8.47	11.42	13.12	17.80	20.16	......	......
.33	3 15/16	1.38	2.08	2.79	5.65	8.61	11.59	13.32	18.06	20.46	......	......
.34	4 1/16	1.40	2.11	2.83	5.74	8.73	11.76	13.52	18.32	20.75	......	......
.35	4 3/16	1.42	2.15	2.87	5.82	8.86	11.93	13.71	18.58	21.04	......	......
.36	4 5/16	1.44	2.18	2.91	5.90	8.98	12.09	13.90	18.83	......	......	......
.37	4 7/16	1.46	2.21	2.95	5.98	9.10	12.26	14.09	19.08	......	......	......
.38	4 9/16	1.48	2.24	2.99	6.07	9.22	12.42	14.27	19.33	......	......	......
.39	4 11/16	1.50	2.27	3.03	6.14	9.34	12.58	14.46	19.57	......	......	......
.40	4 13/16	1.52	2.29	3.07	6.22	9.46	12.73	14.64	19.81	......	......	......
.41	4 15/16	1.54	2.32	3.11	6.30	9.58	12.88	14.82	......	......	......	......
.42	5 1/16	1.56	2.35	3.15	6.38	9.69	13.04	14.99	......	......	......	......
.43	5 3/16	1.58	2.38	3.18	6.45	9.80	13.19	15.17	......	......	......	......
.44	5 1/4	1.60	2.41	3.22	6.53	9.91	13.33	15.34	......	......	......	......
.45	5 3/8	1.62	2.43	3.26	6.60	10.03	13.48	15.51	......	......	......	......
.46	5 1/2	1.63	2.46	3.29	6.68	10.13	13.62	15.68	......	......	......	......
.47	5 5/8	1.65	2.49	3.33	6.75	10.24	13.77	15.84	......	......	......	......
.48	5 3/4	1.67	2.51	3.36	6.82	10.35	13.91	16.01	......	......	......	......
.49	5 7/8	1.69	2.54	3.40	6.89	10.46	14.05	16.17	......	......	......	......
.50	6 0	1.71	2.57	3.43	6.96	10.56	14.19	16.33	......	......	......	......
.51	6 1/8	1.72	2.59	3.47	7.03	10.66	14.33	......	......	......	......	......
.52	6 1/4	1.74	2.62	3.50	7.10	10.77	14.46	......	......	......	......	......
.53	6 3/8	1.76	2.64	3.53	7.17	10.87	14.60	......	......	......	......	......
.54	6 1/2	1.77	2.67	3.57	7.24	10.97	14.73	......	......	......	......	......
.55	6 5/8	1.79	2.69	3.60	7.31	11.07	14.86	......	......	......	......	......
.56	6 3/4	1.81	2.72	3.63	7.37	11.17	14.99	......	......	......	......	......
.57	6 13/16	1.82	2.74	3.66	7.44	11.27	15.12	......	......	......	......	......
.58	6 15/16	1.84	2.77	3.70	7.50	11.36	15.25	......	......	......	......	......
.59	7 1/16	1.85	2.79	3.73	7.57	11.46	15.38	......	......	......	......	......
.60	7 3/16	1.87	2.81	3.76	7.63	11.55	15.50	......	......	......	......	......
.61	7 5/16	1.89	2.84	3.79	7.70	11.65	15.63	......	......	......	......	......
.62	7 7/16	1.90	2.86	3.82	7.76	11.74	15.75	......	......	......	......	......
.63	7 9/16	1.92	2.88	3.85	7.82	11.83	15.88	......	......	......	......	......
.64	7 11/16	1.93	2.91	3.88	7.88	11.93	16.00	......	......	......	......	......
.65	7 13/16	1.95	2.93	3.91	7.95	12.02	16.12	......	......	......	......	......
.66	7 15/16	1.96	2.95	3.94	8.01	12.11	16.24	......	......	......	......	......
.67	8 1/16	1.98	2.97	3.97	8.07	12.20	16.36	......	......	......	......	......
.68	8 3/16	1.99	3.00	4.00	8.13	12.29	16.48	......	......	......	......	......
.69	8 1/4	2.01	3.02	4.03	8.19	12.38	16.59	......	......	......	......	......
.70	8 3/8	2.02	3.04	4.06	8.25	12.46	16.71	......	......	......	......	......
.71	8 1/2	2.04	3.06	4.09	8.31	12.55	16.83	......	......	......	......	......
.72	8 5/8	2.05	3.08	4.12	8.37	12.64	16.94	......	......	......	......	......
.73	8 3/4	2.07	3.11	4.15	8.42	12.73	17.06	......	......	......	......	......
.74	8 7/8	2.08	3.13	4.17	8.48	12.81	17.17	......	......	......	......	......
.75	9 0	2.10	3.15	4.20	8.54	12.90	17.28	......	......	......	......	......

The agreement of the discharge formula with the experimental data is shown in Table IV to be within a mean of approximately 0.5 per cent, but there are a few individual points more than 1 per cent off.

TABLE IV.—*Difference between discharge obtained by experiment and as computed from the formula*

Size of orifice.	Difference of heads.	Discharge (cubic feet per second). From experiment.	Discharge (cubic feet per second). From average curve.	Discharge (cubic feet per second). Computed from formula.	Experimental discharge compared with curve. Difference (cubic feet per second).	Experimental discharge compared with curve. Per cent.	Computed discharge compared with curve. Difference (cubic feet per second).	Computed discharge compared with curve. Per cent.
Square feet.	*Feet.*							
0.5 by 1.0 foot (exact area, 0.4997 square foot)	0.1	0.756	0.756	0.751	0.000	0.00	−0.005	−0.66
	.4	1.525	1.525	1.521	.000	.00	− .004	− .26
	.6	1.876	1.873	1.869	+.003	+ .16	− .004	− .21
	.8	2.163	2.167	2.164	−.004	− .18	− .003	− .14
0.5 by 1.5 feet (exact area, 0.7521 square foot)	.1	1.141	1.141	1.140	.000	.00	− .001	− .09
	.3	1.987	1.983	1.986	+.005	+ .25	+ .004	+ .20
	.5	2.548	2.559	2.571	−.011	− .43	+ .012	+ .47
	.8	3.239	3.237	3.259	+.002	+ .06	+ .022	+ .68
0.5 by 2.0 feet (exact area, 0.9996 square foot)	.1	1.511	1.517	1.530	−.006	− .40	+ .013	+ .86
	.3	2.636	2.638	2.652	−.002	− .08	+ .014	+ .53
	.5	3.404	3.407	3.426	−.003	− .09	+ .019	+ .56
	.8	4.318	4.314	4.335	+.004	+ .09	+ .021	+ .49
1.0 by 2.0 feet (exact area, 2.0001 square feet)	.1	3.108	3.114	3.100	−.006	− .18	− .014	− .45
	.3	5.407	5.398	5.386	+.009	+ .17	− .012	− .22
	.5	6.969	6.966	6.964	+.003	+ .04	− .002	− .03
	.8	8.814	8.815	8.821	−.001	− .01	+ .006	+ .07
1.0 by 3.0 feet (exact area, 2.9997 square feet)	.1	4.785	4.781	4.776	+.004	+ .08	− .005	− .10
	.3	8.158	8.170	8.209	−.012	− .15	+ .039	+ .48
	.5	10.471	10.476	10.559	−.005	− .05	+ .083	+ .79
	.8	13.186	13.186	13.312	.000	.00	+ .126	+ .96
1.0 by 4.0 feet (exact area, 3.9970 square feet)	.1	6.505	6.471	6.486	+.034	+ .52	+ .015	+ .23
	.2	8.983	9.062	9.085	−.079	− .97	+ .023	+ .25
	.4	12.682	12.688	12.722	−.006	− .05	+ .034	+ .27
	.7	16.603	16.661	16.699	−.058	− .35	+ .038	+ .23
1.5 by 3.0 feet (exact area, 4.4998 square feet)	.1	7.428	7.430	7.411	−.002	− .03	− .019	− .26
	.2	10.507	10.471	10.414	+.036	+ .34	− .057	− .54
	.4	14.888	14.750	14.638	+.138	+ .94	− .112	− .76
	.5	16.488	16.474	16.332	+.014	+ .09	− .142	− .86
	.6	18.103	18.030	17.863	+.073	+ .40	− .167	− .93
1.5 by 4.0 feet (exact area, 6.0006 square feet)	.10	10.155	10.170	10.172	−.015	− .15	+ .002	+ .02
	.15	12.435	12.365	12.361	+.070	+ .57	− .004	− .03
	.20	14.275	14.181	14.196	+.094	+ .66	+ .015	+ .11
	.25	15.741	15.813	15.803	−.072	− .45	− .010	− .06
	.30	17.313	17.258	17.253	+.055	+ .32	− .005	− .03
	.339	18.398	18.302	18.297	+.096	+ .52	− .005	− .03
	.363	18.919	18.915	18.909	+.004	+ .02	− .006	− .03
1.5 by 4.5 feet (exact area, 6.7513 square feet)	.1	11.356	11.479	11.576	−.123	−1.07	+ .097	+ .85
	.2	15.978	15.993	16.114	−.015	− .09	+ .121	+ .76
	.25	17.865	17.791	17.922	+.074	+ .42	+ .131	+ .74
	.283	18.818	18.880	19.016	−.062	− .33	+ .136	+ .72
2.0 by 4.0 feet (exact area, 8.0065 square feet)	.05	10.512	10.407	10.239	+.105	+1.01	− .168	−1.61
	.10	14.498	14.481	14.241	+.017	+ .12	− .240	−1.66
	.10	14.398	14.481	14.241	−.083	− .57	− .240	−1.66
	.15	17.671	17.571	17.270	+.100	+ .57	− .301	−1.71
	.15	17.810	17.571	17.270	+.239	+1.36	− .301	−1.71
2.0 by 4.5 feet (exact area, 8.9990 square feet)	.05	11.710	11.714	11.722	−.004	− .03	+ .008	+ .07
	.10	16.368	16.379	16.249	−.011	− .07	− .130	− .79
	.125	18.276	18.260	18.047	+.016	+ .09	− .213	−1.17

The computed discharges agree with the discharges taken from the experimental curves within a maximum error of less than 1 per cent, except on the 2 by 4 foot and the 2 by 4.5 foot orifices, where the experimental control was not entirely dependable and somewhat in error.

The computed discharges agree with the experimental discharge data within 1 per cent, except for the large orifices noted above. It is therefore safe to assume that this type of orifice and the discharge formula for it will give results within 2 per cent of the truth for all cases, and probably within 1 per cent for the majority of cases.

Notwithstanding the fact that this type of orifice will permit a measurement of flow with an accuracy well within practical demands and has other previously enumerated practical advantages, it must be borne in mind that a correction factor will have to be applied to the tables unless the depth of water in the channel of approach is 2.5 feet, which is the depth upon which the formula and tables are based. Such correction factors are not only bothersome, but often are a source of grave error; and therefore it is desirable to maintain the standard depth of water if possible. The correction factor is made necessary for a change in depth of water in the channel of approach because of a changed velocity of approach and also because of a changed contraction distance at the top of the orifice. Where the standard depth of water, 2.5 feet, can not be obtained, or where there is a considerable fluctuation in the depth of water, the use of the modified submerged orifice should be discouraged.

SUPPLEMENTAL TESTS ON SUBMERGED ORIFICES

Some unusual results were obtained from the orifice experiments made in the hydraulic laboratory in the summer of 1914, and because of the revolutionary character of those original data, special care has been taken to insure their accuracy. Every part of the apparatus and every phase of the results that offered a very probable source of error have been questioned and examined. The data were consistent with themselves, but did not conform to the somewhat arbitrary theory that has grown piecemeal from isolated parts of experiments. It was thought probable that the gate placed at the end of the channel of recession to control the submergence on the orifice was so close to the orifice as to have a marked effect upon the flow through the orifice.

A series of control experiments was made during the summer of 1916, in which the orifice structures were duplicates of those used in the original experiments, the heads were the same, the same general methods of experimentation were used; but the 1916 check experiments were made in the concrete rating channel outside the laboratory. The orifice structure was placed near the middle of the channel, which is 200 feet long, 5 feet wide, and 3.5 feet deep, and there is no gate or other obstruction in the channel within approximately 100 feet of the orifice. Although it

was not possible to control the flow of water as perfectly in the long channel as in the laboratory proper, still the regulation of the head was quite good, and in all experiments the discharge was determined volumetrically.

Of the 22 experiments made for check purposes, 15 were comparable directly with original experiments made in 1914. They show that the gate caused an increase in the discharge, but, when allowance is made for probable experimental error, this increase is a maximum of approximately 2 per cent. The 1916 experimental results do not show the original data to have been in error an impractical amount due to the close proximity of the control gate to the orifice.

THE VENTURI FLUME

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INTRODUCTION

Many devices have been developed for the measurement of water under field conditions—for example, in its delivery to irrigators. Nearly all of these devices employ the principles of either the weir or the orifice and, though each device is adapted to use in certain localities, probably none works satisfactorily under a great variety of field conditions. The ideal measuring device would (1) be inexpensive to construct, (2) be simple to operate, (3) require little maintenance, (4) be free from working parts, (5) be accurate in its measurement, (6) be free from sand, silt, or floating-trash troubles, and (7) require but little loss of head in the ditch. Such a panacea for all measurement-of-water ills does not seem probable, but progress is undoubtedly being made toward that end. The type of flume tested in the experiments on which this report is based possesses many of the qualities enumerated and may prove to be a satisfactory measuring device under general field conditions.

The purpose of this article is to present the fundamental plans and results of preliminary experiments on a new type of device, called the "Venturi flume," for measuring water in open channels, in order that those in practical need of such a device may know of its existence. Furthermore, it is hoped that the construction of larger sizes of Venturi flumes than were tested in the laboratory will be encouraged thereby and that they can be calibrated. It is not probable that the last word has been said on the design of the Venturi flume, for, although it has considerable promise, changes in details may prove to be necessary. The laboratory and field tests made thus far have failed to develop any serious inherent defects in the device.

Experiments made in the hydraulic laboratory at Fort Collins, Colorado, on measuring devices led to the development of the so-called Venturi flume during the season of 1915. It consists essentially of a flume with a converging and a diverging section and short "throat" section between them. The floor, which is level, is placed at the elevation of the bottom of the channel in which it is set. After many experiments had been made with different forms and shapes, the designs shown in figures 1, 6, 7, and 11 were adopted as most nearly meeting practical requirements. Venturi flumes with rectangular and trapezoidal cross sections (fig. 1, 6) no doubt will be the most used, but the other types (fig. 7, 11) were designed to meet special conditions where small flows must be measured.

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The action of this device depends upon an adaptation or extension of Venturi's principle to the flow of a liquid in an open channel. As water passes through the flume there is a slight surface slope in the converging section, a rather sudden depression in the "throat" section, and a rise in the diverging section. The actual loss of head is small. The determination of the flow depends upon the velocity and wetted cross-sectional area at two points in the flume, and two gage readings, therefore, are necessary. One gage has been arbitrarily located upstream from the throat a distance equal to two-thirds the length of the converging section, to avoid possible influence due to contraction currents nearer the entrance to the flume; and the other gage has been located at the middle of the throat section, in order to obtain the greatest possible difference in elevation of water surface. The zero of these gages must be at the elevation of the floor of the flume, and it is especially important that the zero of the gages be at exactly the same elevation. The difference in heads, H_d, is a more important factor in determining the discharge than the depth of water in the channel, H_a or H_b.

Still boxes, or gage wells, are necessary for accurate readings of the water levels, because of the comparatively high velocity of the water flowing through the structure. Field tests on small Venturi flumes[1] indicated that readings taken to the nearest 0.01 foot on staff gages placed at the proper locations inside the flume, with the face of the gages countersunk flush with the surface of the side of the flume, would give an accuracy of measurement sufficient for general purposes. This would overcome the necessity for using gage wells, but recent tests made in the laboratory show that such staff-gage readings do not agree with readings taken in the gage wells when there is enough fall in the carrying channel to give a high velocity of flow through the flume, in which case H_d is a considerable amount. Until more is known of the accuracy of gages under different arrangements, caution should be used.

Instrument makers are at work on an automatic register to make graphs of the water elevations at the two gages, both records to appear on a single sheet. An integrating register would be most desirable, but the complexity of the law of flow through the flume certainly would require a complicated instrument.

The effect of the velocity of approach is automatically cared for in the device, and the formula takes account of the velocity of the water at each gage. The experiments indicate that the Venturi flume will be free from interference due to changes in the canal section, such as occur often from sand or silt accumulations or aquatic growths. Such obstructions make the use of the ordinary rating flume very troublesome, if not quite impossible, but these obstructions result only in changing the relative gage readings of the Venturi flume without altering the calibration of

[1] Tests made on the North Platte Project, United States Reclamation Service, Mitchell, Nebraska, under the general direction of Mr. Andrew Weiss, Project Manager.

the device. Since the velocity increases throughout the converging section, all material carried into the flume also will be carried out, and this self-cleaning feature is of considerable practical importance. When the depth of water is low, floating trash might lodge in the throat of the V-notch Venturi flume, which is of small cross section, but it would cause an accumulation of water in the upstream channel until the wetted cross section at the throat would be sufficient to allow the obstruction to pass. It must be borne in mind that a Venturi flume of whatever form must not be placed below canal grade, for this would give a standing-water condition which would alter the calibration of the device, and it would also allow sand and silt to accumulate within the structure at low velocities. It is important also that the width of the channel of approach be not greatly in excess of the greatest width of the flume, as this permits a silt bank to be deposited at either side wing of the flume.

A desirable phase of this device is the practical connection which it may make with the ditch banks. At the ends of the structure, wings may be placed at an angle of 90° to the axis of the structure to make the connection with the ditch banks, or the ends of the structure may be joined directly to the ditch lining.

Another practical feature in connection with the Venturi flume is the small loss of head required for purposes of measuring the flow. Table I shows for the V-notch flume the lost head for the different discharges obtained with different depths of water. The head at the upstream gage is called H_a, the head at the throat gage is called H_b, and the difference between these heads $(H_a - H_b)$ is called H_d. Under usual conditions of operation the lost head will be negligible.

TABLE I.—*Loss in head (in feet) in V-notch Venturi flume for different heads at the two gages*

H_a	H_d=0.05		H_d=0.10		H_d=0.15		H_d=0.20		H_d=0.25		H_d=0.30	
	Q in second-feet.	Loss in head in feet.	Q in second-feet.	Loss in head in feet.	Q in second-feet.	Loss in head in feet.	Q in second-feet.	Loss in head in feet.	Q in second-feet.	Loss in head in feet.	Q in second-feet.	Loss in head in feet.
0.4	0.10	0.06	0.11	0.08								
.6	.26	.05	.30	.07								
.8	.49	.04	.60	.06	0.62	0.15						
1.0	.81	.03	1.00	.06	1.09	.13	1.12	0.24				
1.2	1.20	.03	1.52	.06	1.63	.11	1.76	.17	1.79	0.29		
1.4	1.71	.03	2.17	.05	2.42	.10	2.56	.14	2.63	.21	2.67	0.31
1.6	2.33	.03	2.96	.05	3.31	.09	3.52	.12	3.65	.17	3.74	.23

RECTANGULAR VENTURI FLUME

The original idea was to invent a device which would replace the ordinary rating flume, such as is used in irrigation canals. It was thought that the flume might be converted into a self-contained measuring device by placing a restricted section in the flume, which would cause a loss of

head, and a determination of such loss of head would indicate the volume of water flowing in the channel. Thus far, Venturi's principle had not been considered in the action of such a device. Small flumes with vertical sides were used in the preliminary experiments; and, after employing several different ratios of widths of throat to lengths of flume, lengths

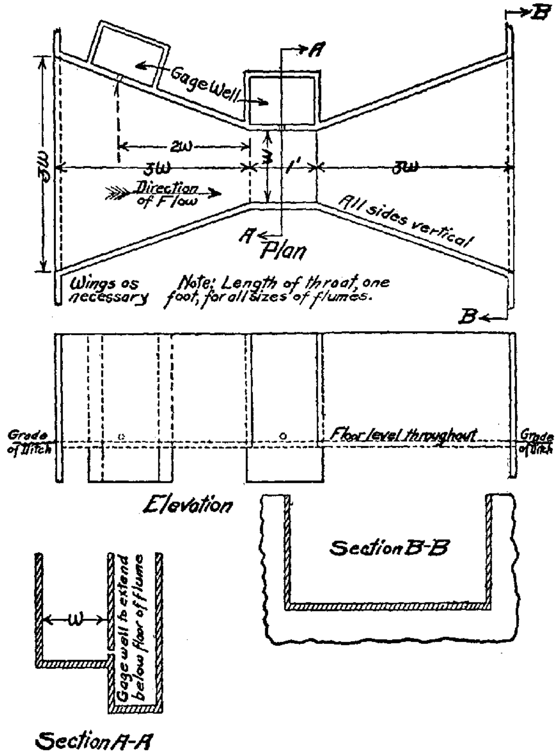
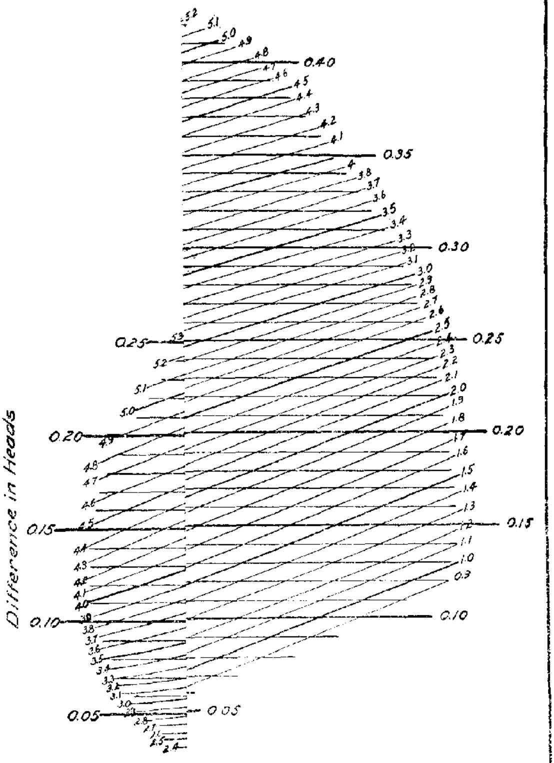


FIG. 1.—Standard plans for the Venturi flume with rectangular cross section.

of throat, and arrangements of gages and end wings, the form shown in figure 1 was chosen as the standard. A greater length of converging and diverging section and a rounding of the throat section would result in less loss of head and greater accuracy in measurement of flow, but the standard was chosen as a compromise between accuracy and cost.



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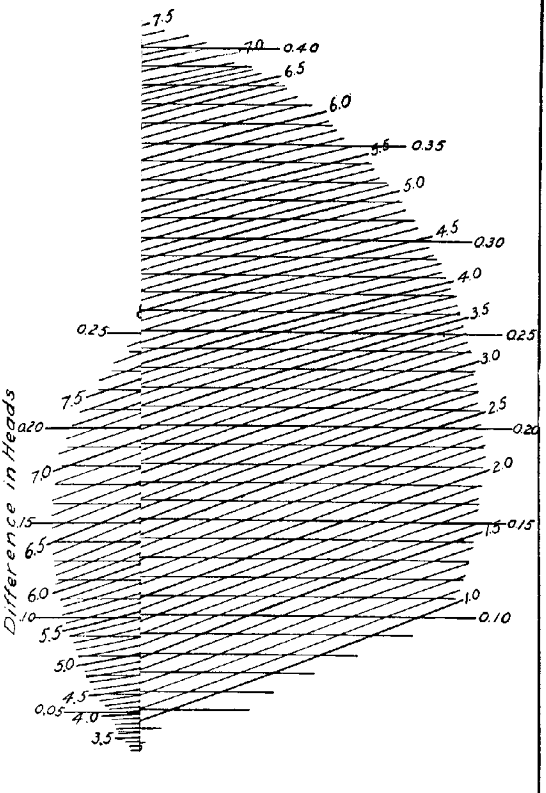

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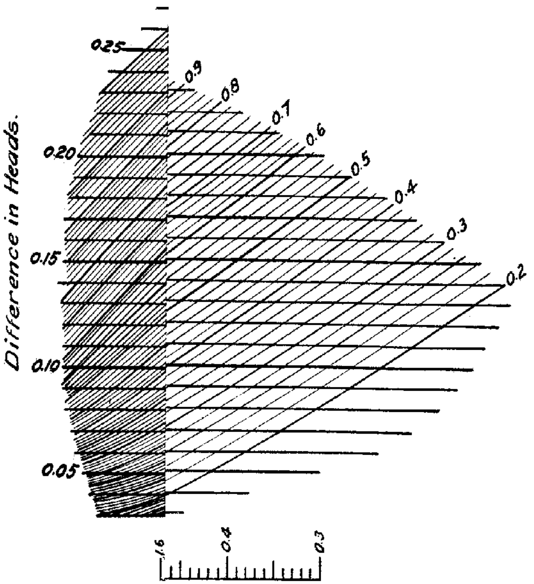

'raight-edge across the two
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the two gage readings.

The Venturi flume with rectangular cross section is especially simple to build of any material suitable for use in water, and will probably be the most popular type. Its practical minimum throat width is 1 foot, and the largest one thus far constructed has a throat width of 7 feet.

A general formula for the discharge through rectangular Venturi flumes has not been worked out, because calibrations have not been made on flumes large enough to warrant a formula of general application. Discharge curves are given in figures 2, 3, and 4 for throat widths of 1, 1½, and 2 feet.

TRAPEZOIDAL VENTURI FLUME WITH SIDE SLOPES OF 1½ TO 1

Although no trapezoidal Venturi flumes with side slopes of 1½ to 1 have been constructed, there is no reason apparent why their behavior would not be similar to that of rectangular cross-sectional type. The side slopes of 1½ to 1 will fit the majority of canal banks, and the resulting cross section will accommodate a greater range of discharges than the rectangular flumes. Therefore it is believed that, for the larger canals, the more satisfactory type of Venturi flume will have a trapezoidal cross section with side slopes of 1½ to 1. It will fit nicely with concrete lining of canals. This form does not call for warped surfaces, because the slopes are taken normal to the axial line of the flume, which is in a plane normal to the side of the throat section but is not normal to the side of the converging and diverging sections. The general plans for this type are given in figure 6, but no discharge curves are available at this time. It is expected that calibrations will be made from structures as they are installed under actual field conditions.

V-NOTCH VENTURI FLUME

There has been a demand for many years for a device to measure small flows of water where the permissible loss of head is small, or where sand and silt is carried by the water. After repeated unsuccessful attempts had been made to arrange a modification of an orifice or weir to meet these conditions, it was decided to ascertain what combination could be made of the Venturi flume and the triangular-notch weir. The result was the V-notch Venturi flume shown in figures 7 and 8. The side slopes of ½ to 1, in a plane normal to the axis of the flume, give a cross-sectional area of the throat section for different depths of water, which allows a good range of discharge from extreme high to low heads. This form is applicable under conditions of head commonly found in small ditches to flows of from 0.1 to 2 or 3 second-feet.

Discharges through V-notch Venturi flumes are given in graphic form in figure 5, and those computed from the formula are given in Table II.

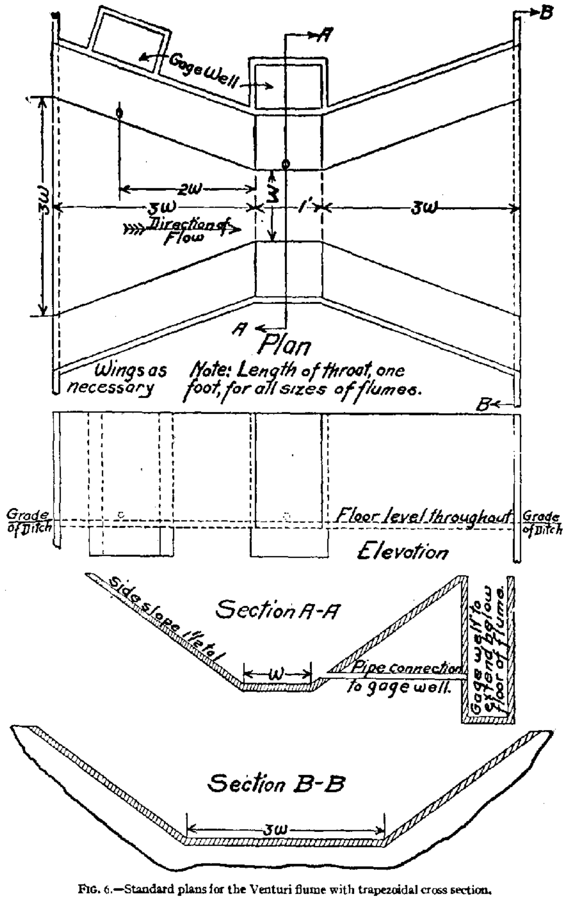

FIG. 6.—Standard plans for the Venturi flume with trapezoidal cross section.

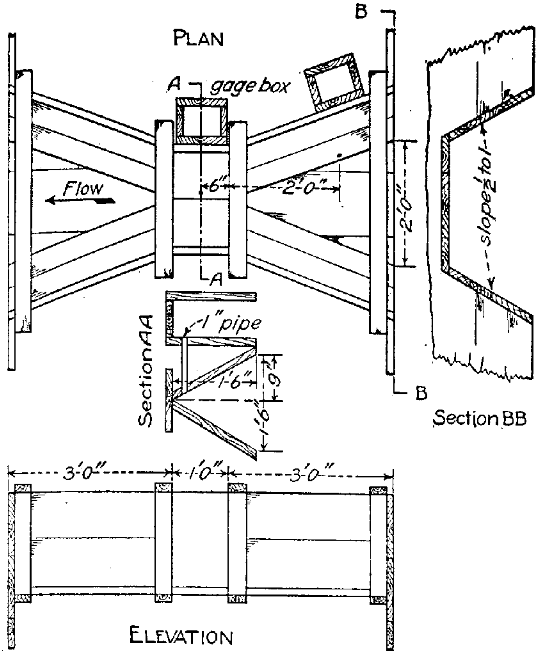

Fig. 7.—Plan, elevation, and sections of the V-notch Venturi flume.

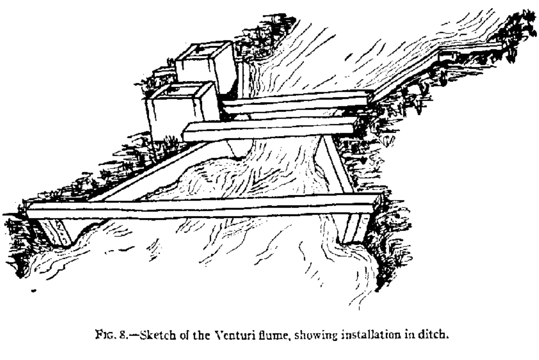

Fig. 8.—Sketch of the Venturi flume, showing installation in ditch.

The effect upon the discharge caused by different arrangements of the channels of approach and recession is shown by the following experimental results. The discharges for each condition have been compared with those for the standard arrangement.

Extending the converging section to a length of approximately 6 feet instead of 3 feet, as in the standard plan (fig. 7), but with the same angle of convergence, caused a decrease in discharge of not to exceed 0.5 per cent for any depth of water.

A channel of approach with parallel sides, having a side slope of ½ to 1 and a bottom width of 2 feet, joined to the upstream end of the Venturi flume caused a decrease in discharge of less than 1 per cent for any depth of water. This change was comparable to eliminating the 90° wings at the upstream end and joining the device directly to the lined section of a ditch.

With the standard construction for the upstream portion of the flume, a channel of recession similar to the previously described channel of approach was provided. This change had no appreciable effect upon the discharge for any depth of water.

A piece of 2- by 4-inch timber was placed on edge at the upstream end of the flume and nailed to the floor. Its position was normal to the axis of the flume, and it extended across the full width of the section. The increase in discharge due to this change did not exceed 1 per cent for any depth of water.

DERIVATION OF FORMULA FOR DISCHARGE THROUGH THE V-NOTCH VENTURI FLUME

From Bernoulli's theorem:

$$\frac{V_a^2}{2g}+p+H_a=\frac{V_b^2}{2g}+p+H_b \qquad (1)$$

n which V_a and H_a represent the velocity and head at the gage in the upstream section and V_b and H_b represent the velocity and head in the throat section.

from (1) $V_b^2=V_a^2+2gH_d$ (2)

where $H_d=H_a-H_b$

$$Q=A_aV_a=A_bV_b$$

$$V_a=\frac{A_bV_b}{A_a}=\frac{\dfrac{H_b^2V_b}{2}}{(2⅔+H_a)\dfrac{H_a}{2}} \qquad (3)$$

substituting (3) in (2)

$$V_b^2=\frac{H_b^4V_b^2}{(2⅔+H_a)^2H_a^2}+2gH_d$$

$$V_b=\sqrt{\frac{2gH_d}{1-\dfrac{H_b^4}{(2⅔+H_a)^2H_a^2}}}$$

and

$$Q=V_bA_b=\frac{H_b^2}{2}\sqrt{\frac{2gH_d}{1-\dfrac{H_b^4}{(2⅔+H_a)^2H_a^2}}} \qquad (4)$$

As shown in Table III, discharge values computed by equation (4) are higher than those obtained by experiment, because the equation does not contain a correction factor for the effect of contraction and friction. Table III and figure 9, plotted from this table, show the correction factor (C) to be greater for high and low values of H_d than for medium values of H_d, and (C) increases as H_a increases. To avoid confusion, coefficients (C) for H_a's of 1.6 feet, 1 foot, and 0.4 foot, only, have been plotted in figure 9. The assumed limiting curves are shown in dotted

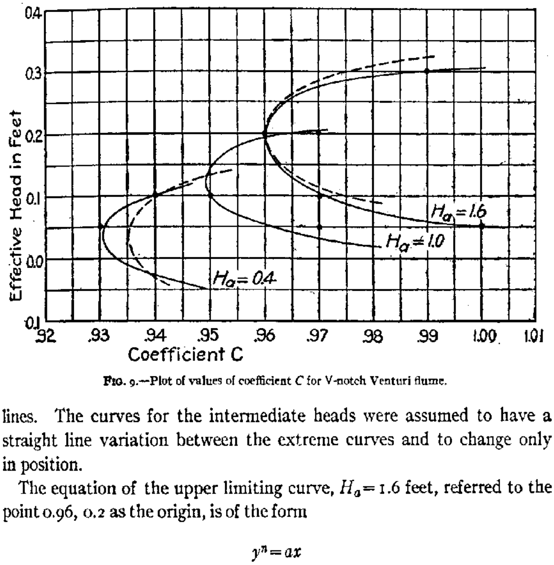

FIG. 9.—Plot of values of coefficient C for V-notch Venturi flume.

lines. The curves for the intermediate heads were assumed to have a straight line variation between the extreme curves and to change only in position.

The equation of the upper limiting curve, $H_a=1.6$ feet, referred to the point 0.96, 0.2 as the origin, is of the form

$$y^n = ax$$

substituting values to find n and a,

$$0.1^n = 0.0167a$$

$$0.03^n = 0.0015a$$

$$\frac{0.1^n}{0.03^n} = \frac{0.0167a}{0.0015a}$$

$$3.33^n = 11.13$$

$$n \log 3.33 = \log 11.13$$

$$n = \frac{\log 11.13}{\log 3.33}$$

$$n = 2.00$$

$$(0.1)^2 = 0.0167a$$

$$a = \frac{0.01}{0.0167} = 0.6$$

The equation is therefore $y^2 = 0.6\,x$. With the origin moved to the point *o*, *o*, the equation of the upper limiting parabola becomes

$$(y-0.2)^2 = 0.6(x-0.96) \qquad (5)$$

and similarly the equation for the lower limiting curve, H_a, approxi-

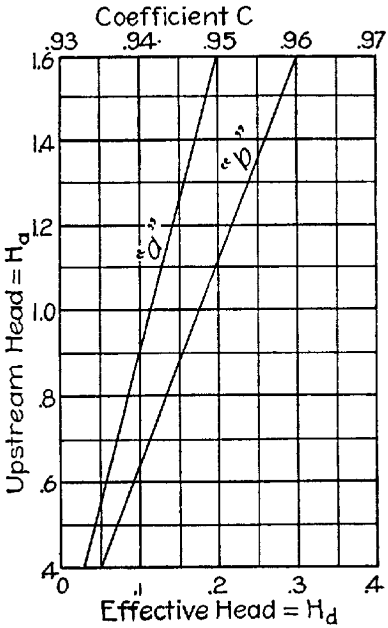

FIG. 10.—Plot of constants for curves shown in figure 9.

mately 0.4 foot, is found to be

$$(y-0.03)^2 = 0.6(x-0.935) \qquad (6)$$

The constants for each curve were assumed to take a straight line variation, as shown in figure 10, in which the *a* line is for the constants with *y*, and the *b* line is for the constants with *x*.

By proportion from figure 10 and the substitution H_a for *y*,

$$a = 0.14y - 0.02$$

or

$$a = 0.14H_a - 0.02$$

and

$$b = 0.02H_a + 0.93$$

Substituting these values in equation (5) or (6) gives

$$(y-0.14H_a+0.02)^2=0.6(x-0.02H_a-0.93)$$

but since $y=H_d$ and $x=c$,

$$(H_d-0.14H_a+0.02)^2=0.6(c-0.02H_a-0.93)$$

whence

$$C=\frac{(H_d-0.14H_a+0.02)^2+0.01H_a+0.56}{0.6}$$

After the combination of the value of the coefficient C with the theoretical formula, equation (4) becomes

$$Q=\left[\frac{(H_d-0.14H_a+0.02)^2+0.01H_a+0.56}{0.6}\right]\frac{H_b^2}{2}\sqrt{\frac{2g\,H_d}{1-\frac{H_b^4}{(2\frac{2}{3}+H_a)^2H_a^2}}} \quad (7)$$

in which the bracketed portion represents the coefficient for contraction and friction, $\frac{H_b^2}{2}$ represents the wetted cross section of the throat of the flume, and the radical expression represents the velocity of flow.

Simplifying the above equation gives

$$Q=6.68H_b^2[(H_d-0.14H_a+0.02)^2+0.01H_a+0.56]\sqrt{\frac{H_d}{1-\frac{H_b^4}{(2\frac{2}{3}+H_a)^2H_a^2}}} \quad (8)$$

which is the discharge formula for the V-notch Venturi flume. Table II has been computed for this equation. The experimental discharge values are shown in curve form in figure 5.

Discharge values computed from equation (8), for any given H_a, increase as H_d is increased up to a certain point; but with further increase of H_d the discharge values decrease. At first thought this seems to be impossible; but it must be true, because in the limiting case where $H_d=H_a$, H_b becomes zero, and from the formula, Q must equal zero. Discharge values computed from equation (8) must plot into smoothly continuous curves of a reversed character, and these values must therefore ultimately decrease. From equation (7) it is evident that the wetted cross-sectional area of the throat varies as the square of the head, H_b, while the velocity varies nearly as the square root of the difference in head, H_d, and therefore for any given H_a, as the H_d increases the area decreases more rapidly than the velocity increases.

The calibration experiments with the V-notch Venturi flume did not show any decrease in discharge, such as mentioned above. For each H_a there is a definite limit to the value of the H_d which may be obtained

in the practical operation of this device, and it is probable that this limit is about at the reversing point on the discharge curves made from the formula. It was found by experiment that for any given H_a after a certain H_b had been obtained, a further lowering of the water surface in the diverging section had no influence upon the elevation of the water at the throat gage, H_b.

TABLE III.—*Comparison of theoretical and experimental discharge values of V-notch Venturi flume*

H_a	H_b	H_d	Q		C
			Experimental.	Computed.[a]	
0.4	0.35	0.05	0.102	0.110	0.93
.4	.30	.10	.107	.114	.94
1.0	.95	.05	.805	.832	.97
1.0	.90	.10	.999	1.052	.95
1.0	.80	.20	1.124	1.165	.97
1.6	1.55	.05	2.300	2.302	1.00
1.6	1.50	.10	2.925	3.023	.97
1.6	1.40	.20	3.525	3.669	.96
1.6	1.30	.30	3.802	3.830	.99

[a] Discharges computed by equation (4), p. 122.

TRAPEZOIDAL VENTURI FLUME WITH SIDE SLOPES OF 1 TO 1

This is a special type which was developed to meet a condition common in some sections of the irrigated West, where a ditch is used to carry a small head of water for orchard irrigation at one time and a flow of approximately 10 second-feet for alfalfa irrigation at another time. This requires a quite flexible measuring device, and therefore called for the design shown in figure 11. The side slopes for this type of Venturi flume are 1 to 1, and it is expected that it will be built only in the one size; that is, with a 6-inch bottom throat width. The discharges through this Venturi flume are given in graphic form in figure 12.

The discharge through the Venturi flume with trapezoidal cross section, having side slopes of 1 to 1 in a plane normal to the axis of the flume and with a bottom throat width of 6 inches, is represented by the following equation, which was derived in a manner similar to that given for the V-notch Venturi flume:

$$Q=\left[\frac{[(H_d-0.09\,H_a-0.005)^2+0.001\,H_a+0.274]}{0.30}\right](\tfrac{1}{2}+H_b)\,H_b\sqrt{\frac{2gH_d}{1-\dfrac{(\frac{1}{2}+H_b)^2H_b^2}{\left(\frac{11}{6}+H_a\right)H_a^2}}}$$

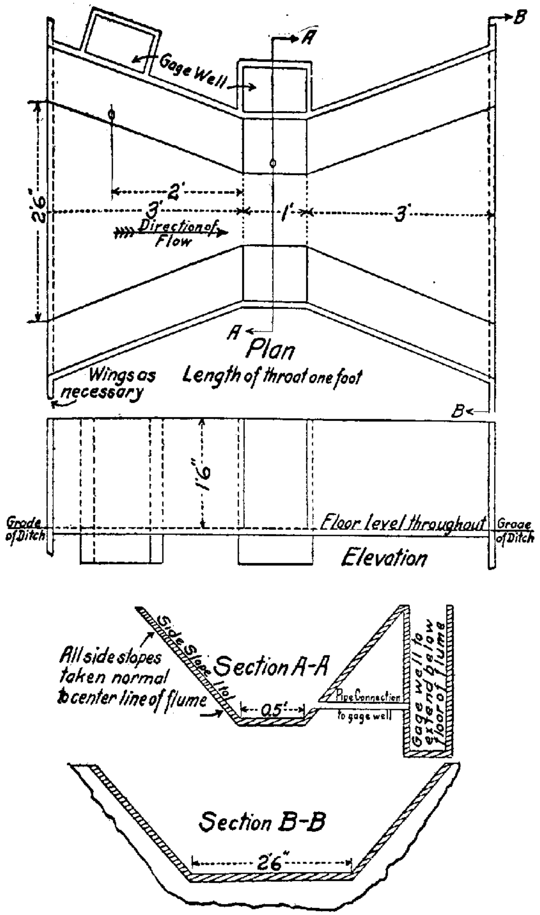

FIG. 11.—Plan for the trapezoidal Venturi flume with 0.5 foot bottom width, side slopes 1 to 1.

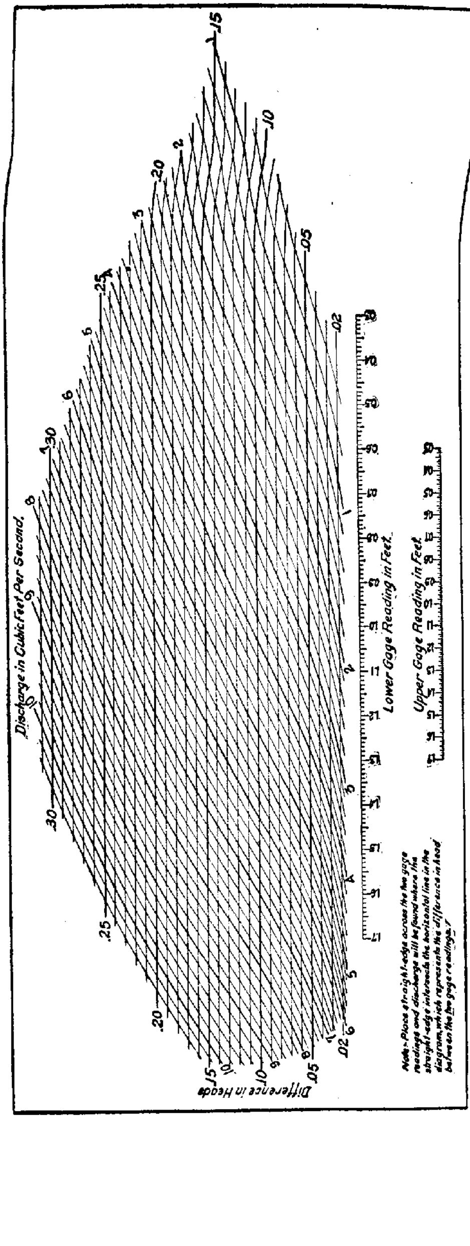

FIG. 12.—Discharge curves for the trapezoidal Venturi flume having 0.5 foot bottom width and side slopes 1 to 1.

CONCLUSION

The Venturi flume is not an exact measuring device, but it is thought to be sufficiently accurate to meet usual practical needs, especially such as are encountered in irrigation practice in the West.

Although experiments have been made only on the smaller sizes of Venturi flumes, it seems reasonable to expect that structures built according to the general plans will be applicable to the measurement of streams of considerable size, with an accuracy compatible with field requirements.

The Venturi flume seems to fulfill the conditions of being free of trouble from sand, silt, or floating trash; requires little loss of head for making the measurement; is a structure that is simple to build, easy to operate, and has a comparatively low cost; and is free from error in measurement due to aquatic growth or other changes in the channel, provided the floor of the flume is not below the grade of the channel.

If the accompanying discharge curves, formulas, or tables are to be used, it is essential that the Venturi flume be built according to the general plans and the gages for measuring the head be placed as shown in the plans. Alterations of the plans or position of gages will necessitate a recalibration for the new arrangement.

A public patent has been applied for which will permit the manufacture or use of this flume by the public without the payment of royalties.

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